Appendix D
Pushover Analysis of Intake Towers

D-1. Introduction

a. This appendix provides an example pushover analysis for a free-standing intake tower that has been analyzed previously to illustrate seismic design procedures in EM 1110-2-2400, response-spectrum mode superposition method in EM 1110-2-6050, and the linear time-history dynamic analysis methodology in EM 1110-2-6051. In this appendix the same free-standing intake tower is used to illustrate the nonlinear static pushover analysis procedures.

b. The nonlinear static pushover analysis or simply pushover analysis is carried out to assess damage vulnerability of structures. A pushover procedure is a series of nonlinear static analyses carried out to develop a capacity curve for the structure. With increasing the magnitude of loading during the pushover analysis, the structural members undergo nonlinear response, and thus weak links and failure modes of the structure are found. The lateral loads representing inertia forces in an earthquake are increased until a target displacement is exceeded or the structure collapses. The target displacement represents the maximum displacement that the structure would likely experience during the design earthquake. The results of pushover analysis are summarized as a plot of lateral load vs. displacement, from which the actual load capacity and ultimate displacement of the structure can be determined.

D-2. Purpose and Objectives

The purpose of this example is to illustrate application of the nonlinear static procedures to pushover analysis of free-standing intake towers. The objectives of the example are:

a. To compute section capacities of the tower using various procedures.

b. To obtain pushover curve showing yielding, cracking, and ultimate displacement of the intake tower.

c. To identify the sequence of plastic hinging and potential failure modes.

D-3. Scope

The scope of this example includes the following:

a. Idealization of the intake tower using various frame elements that account for effects of material inelastic response.

b. Examination of various modeling techniques to identify their strengths and shortcomings.

c. Conducting pushover analyses to obtain capacity curves for various models.

d. Evaluation of results to assess inelastic response behavior and ultimate displacement capacity of the tower.

D-4. Assumptions

The following simplifying assumptions were made to reduce the calculation efforts and to focus on more important aspects of the analysis procedure:

a. Effects of axial load is not considered.

b. Axial force-bending moment (i.e. P-M_x-M_y) interaction is not considered.

c. *P-* effect is not considered.

d. Shear failure is not considered and tower is therefore assumed to fail in flexure.

D-5. Description of Tower

a. Tower Geometry. The intake tower used in this example is described in EM 1110-2-2400. It is a freestanding tower with a height of 60.96 m (200 ft). The tower cross sections vary from 14.63 m %11.28 m (48 ft % 37ft) at the base to 13.41 m % 8.84 m (44 ft % 29 ft) at the top in five steps. The section thicknesses vary from 1.83 m (6 ft) at the base to 0.61 m (2 ft) at the top of the tower. Figure D-1 shows upstream and side elevation views of such a tower. The tower has a 0.61-m- (2.0-ft-) thick concrete slab at the top and a heavy 1.83-m- (6.0-ft-) thick slab at its base. The unit weight of the concrete ($_{conc}$) is 2,403 kg/m^3 (150 lb/ft^3).

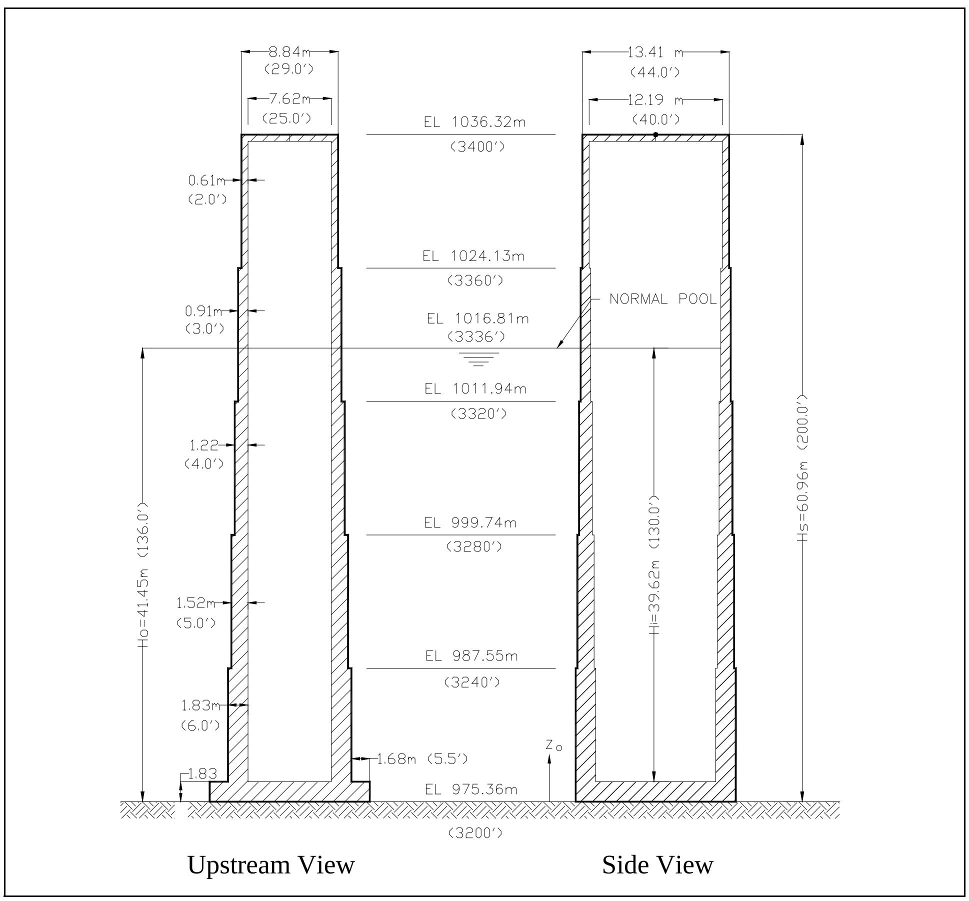


Figure D-1. Example Intake Tower

b. *Material and section properties*. Table D-1 below summarizes the material properties used for the concrete and reinforcing steel. The concrete is assumed to have a compressive strength of 20.7 MPa (3,000 psi) with an ultimate strain of 0.3 percent. The assumed yield strength and ultimate strain for the reinforcing steel are 413.69 MPa (60 ksi) and 5%, respectively. The stress-strain relationship for the concrete and reinforcing steel are discussed later as part of the reinforced concrete fiber element in Paragraph D-8*d*(4). Figure D-2 shows the geometry and reinforcement arrangement at the bottom section of the tower. There is one layer of #11 vertical bars at 30.48-cm (12-inch) spacing along the faces of each wall. Each reinforcement layer is made of 36 bars along the short axis and 47 bars along the long axis of the tower. The section properties including cross-section area, moment of inertia, nominal moment, and cracking moment for pushing along the longitudinal (x) and transverse (y) directions are listed in Table D-2.

Table D-1. Assumed material properties

Parameter		Value	
		Metric Units	English Units
Re-bar Material Properties			
Modulus of Elasisity	(E_s)	199,947.95 MPa	29,000.00 ksi
Specified Yield Strength	(f_y)	413.69 MPa	60.00 ksi
Strain Hardening			0.80 %
Steel Ultimate Stress		517.11	75.00 ksi
Ultimate Strain Hardening			5.00 %
Concrete Matrial Properties			
Modulus of Elasisity	(E_c)	21,525.43 MPa	3,122.00 ksi
Concrete Compressive Strength	(f'_c)	20.68 MPa	3.00 ksi
Modulus of Rupture	(f_r)	2.83 MPa	0.41 ksi
Concrete Ultimate Strain	(ε_c)		0.30 %

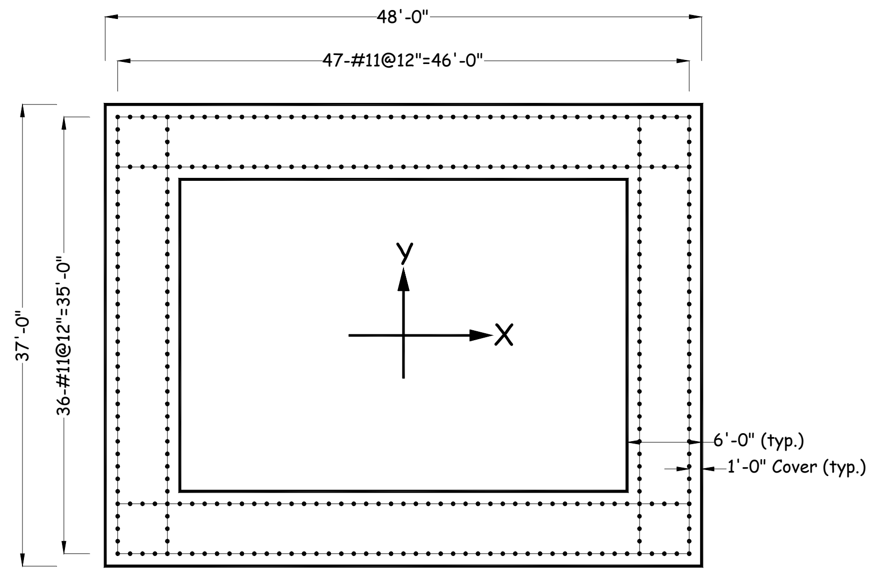


Figure D-2. Geometry and re-bar arrangement at base section of tower

Table D-2. Calculated section properties of the base section shown on Figure 1

For Pushing in X-X direction			
Parameter		Value	
		English Units	Metric Units
Width	(b)	37.00 ft	11.28 m
Depth	(h)	48.00 ft	14.63 m
Cross Section Area	(A)	876.00 ft^2	81.38 m^2
Moment of Inertia	(I_{yy})	243,792 ft^4	2,104.16 m^4
Nominal Moment	(M_{ny})	718,814 k-ft	974.58 N-m
Cracking Moment	(M_{cr})	620,900 k-ft	841.83 N-m
For Pushing in Y-Y direction			
Parameter		Value	
		English Units	Metric Units
Width	(b)	48.00 ft	14.63 m
Depth	(h)	37.00 ft	11.28 m
Cross Section Area	(A)	876.00 ft^2	81.38 m^2
Moment of Inertia	(I_{xx})	155,737 ft^4	1344.16 m^4
Nominal Moment	(M_{nx})	518,879 k-ft	703.51 N-m
Cracking Moment	(M_{cr})	507,900 k-ft	688.62 N-m

D-6. Selection of Analysis Procedures

The following procedures were considered for nonlinear static analysis:

a. Simplified displacement-based analysis. This method involves hand calculations that can be effectively used for the analysis of relatively simple structures such as freestanding towers and bridge piers.

b. Pushover analysis. The pushover analysis is conducted using a load controlled or displacement controlled procedure. Both procedures involve intense calculations requiring the use of computer programs with nonlinear analysis capabilities.

(1) Load-controlled procedure involves incremental application of a monotonic load to the structure until the maximum load is reached or the structure collapses, whichever occurs first. Force control should be used when the magnitude of load is known (such as gravity load), and the structure is expected to support the load.

(2) Displacement-controlled procedure involves incremental application of a monotonic load until the control displacement is reached a pre-specified value or the structure collapses, whichever comes first. Displacement control is used when the value of the applied load is not known in advance, or when the structure is expected to lose strength. Since the final value of earthquake load can not be determined precisely in advance, the displacement-controlled method is employed in this example.

D-7. Simplified displacement-based analysis

Simplified displacement-based analysis for reinforced concrete structures is described in the reference (COE 2001).

When the nominal moment capacity (M_N) is less than 1.2 times the cracking moment (M_{cr}), the plastic hinge length needed for estimation of rotational capacity can be obtained from:

$$l_p = 0.30 f_y (d_b) \qquad \text{(ksi units)}$$

where f_y is yield strength of the reinforcing steel in ksi and d_b is diameter of the reinforcing steel in inches. For the example tower the ratio of nominal moment to cracking moment for both the x and y directions are less than 1.2:

$$\frac{M_{Ny}}{M_{cry}} = \frac{718{,}814}{620{,}900} = 1.16 < 1.20$$

and

$$\frac{M_{Nx}}{M_{crx}} = \frac{518{,}737}{507{,}900} = 1.02 < 1.20$$

Thus the plastic hinge length can be obtained as follows:

$$l_p = 0.30 f_y (d_b) = 0.30(60.0)(1.56) = 28.08\ in = 2.34\ ft$$

The ultimate rotational capacity is estimated from:

$$\theta_u = \phi_u (l_p) = 0.0003125 \cdot (2.34) = 0.000731(rad) = 0.042(\deg)$$

The ultimate displacement capacity in the strong axis direction x-x is estimated using:

$$\delta_u = \frac{11}{40}\phi_y l^2 + (\phi_u - \phi_y) l_p \left(l - \frac{l_p}{l} \right) = \frac{11}{40}(0.0000539)(200)^2 +$$

$$(0.001302 - 0.0000539)(2.34)\left(200 - \frac{2.34}{200} \right)$$

$$\delta_u = 1.18\ ft = 14.16\ in$$

Where:

$$\phi_y = \frac{\varepsilon_y}{0.8h} = \frac{0.00207}{0.8(48)} = 0.0000539$$

and

$$\phi_u = \frac{\varepsilon_u}{0.8h} = \frac{0.05}{0.8(48)} = 0.001302$$

The ultimate displacement ductility is given by:

$$\mu = 1 + 3.64\left(\frac{\phi_u}{\phi_y} - 1 \right)\frac{l_p}{l}\left(1 - 0.5\frac{l_p}{l} \right) = 1 + 3.64 \cdot (24.15 - 1) \cdot \left(\frac{2.34}{200} \right) \cdot \left(1 - 0.5 \cdot \frac{2.34}{200} \right) = 1.98 \approx 2$$

Where:

$$\frac{\phi_u}{\phi_y} = \frac{0.05}{0.00207} = 24.15$$

D-8. Nonlinear Static Procedures Using Computer Program (Pushover)

a. Various computer programs with nonlinear analysis capabilities can be used to perform a pushover analysis. In this example, SAP2000 (1997) and DRAIN-2DX (1994) programs are used to illustrate the displacement-controlled pushover analysis. In each case, a computer model of the tower is developed and subjected to appropriate lateral loads that are increased incrementally until a target displacement is reached or the tower collapses. The lateral load patterns and target displacements are described below.

b. Controlling Node, Lateral Load pattern and Target Displacement

(1) *Controlling node* is a node at which the displacement is computed and monitored. In this example Node 13 at the top of the tower is selected for this purpose (Figure D-3).

(2) *Lateral load pattern* should closely resemble the probable distribution of the earthquake loads. In this example, a load pattern proportional to the fundamental mode shape of the tower is selected. The lateral loads representing the seismic inertia forces are then obtained from production of the fundamental mode shape and the associated mass tributary.

(3) *Target displacement* of the controlling node shall be at least three times of the yield displacement. The yield displacement is the displacement associated with the first yielding of the reinforcing steel.

c. Pushover Analysis using SAP2000

(1) *Structural model.* The basic geometry and section properties defined in Paragraph D-5 are used to develop a SAP2000 model for the pushover analysis. The model is created like any other analysis, except that frame hinges are introduced to model nonlinear response. Plastic hinges are restricted to frame elements only, even though other types of elements can be present in the model. As shown in Figure D-3, the model consists of 13 nodal points and 12 frame elements. The model is fixed at the base nodal point, while other nodes are free with respect to translation and rotation. SAP2000 can handle for both the material nonlinearity and the geometric nonlinearity. As mentioned earlier only the material nonlinearity is considered in this example for simplicity reason and also because axial loads for intake towers are relatively small. The material nonlinearity is specified at discrete, user-defined hinges along the length of frame elements. Since concrete cracking and steel yielding tend to concentrate at the base of the tower, only a single hinge immediately above the bottom slab was included in the model. Figure D-3 provides two views of the model, an extruded view showing 3D geometry of the tower, and a frame view depicting the nodal points and frame elements.

(2) *Moment-rotation relationship.* For pushover analysis, moment-rotation relationship for plastic hinges should be defined. The default moment-rotation relationships available in SAP2000 have been developed for building structural members. They are generally not appropriate for the lightly reinforced hydraulic structures such as the example tower. For this reason the computer program "*M-Phi*" (Ehsani and Marine 1994) is used to develop a moment-curvature for the bottom section of the tower where the plastic hinges will occur (Figure D-2). Then the moment-rotation for the section is obtained by multiplying the curvature by a plastic hinge length. The plastic hinge length is assumed to be 2.34ft, the same as that estimated in the simplified displacement-based analysis in Section D-7. The estimated moment-rotation diagrams for the bottom section with respect to longitudinal (x) and transverse (y) axes of the

tower are shown in Figure D-4. Also shown on this figure are the nominal moments for comparison.

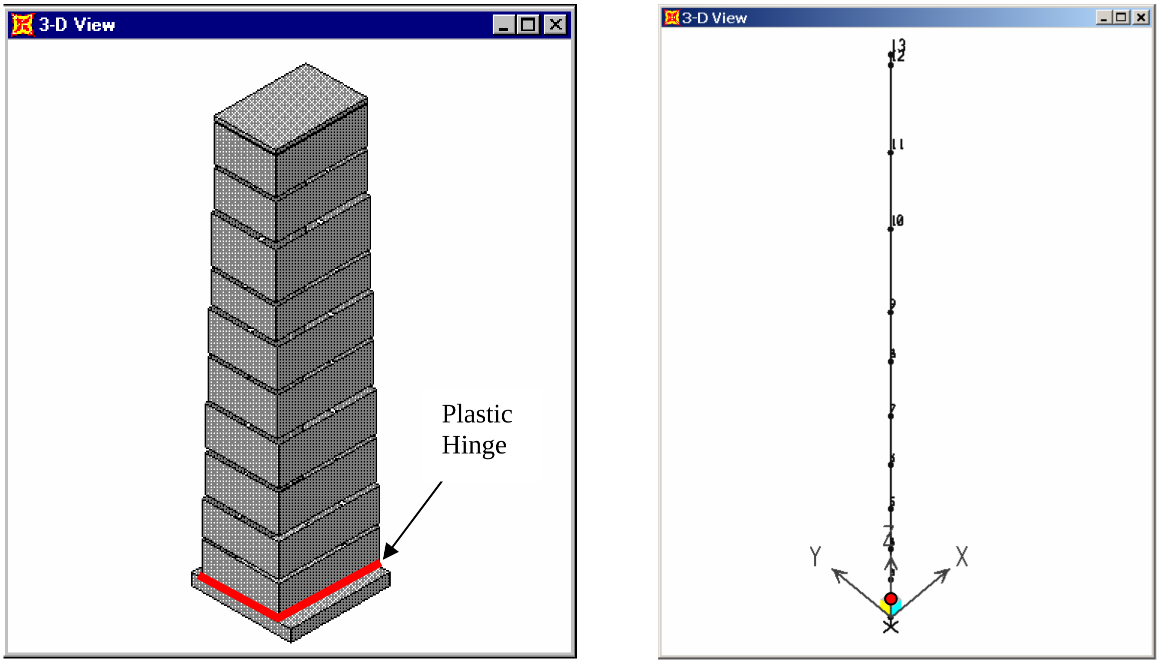


Figure D-3. 3-D view of the sample tower in SAP2000

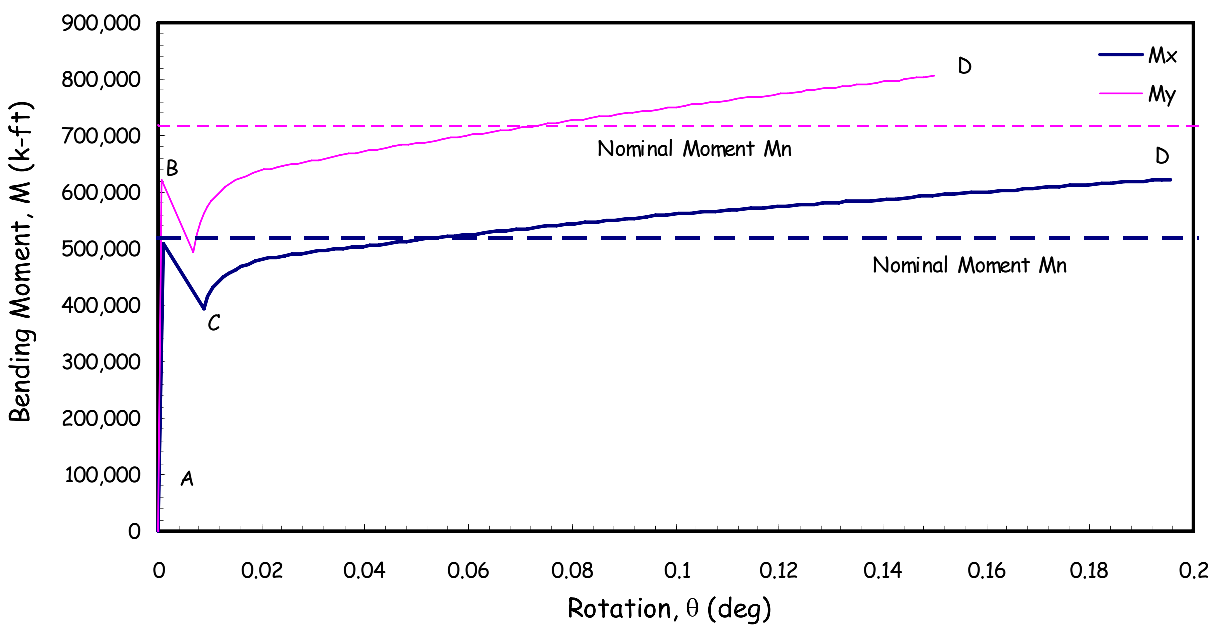


Figure D-4. Moment-rotation relationship for bottom section with l_p= 2.34ft

(3) *Plastic Hinge Properties*. The moment-rotation in Figure D-4 is idealized to define hinge properties for SAP2000 pushover analysis. SAP2000 allows only 4 points to define the hinge properties in accordance with NEHERP (1997). As such the estimated moment-rotation should be idealized according to this restriction. Figure D-5 is a normalized moment-rotation diagram

showing how the four basic points are picked to prepare input for SAP2000. For the lightly-reinforced example tower the ratio of nominal moment to cracking moment is close to one. The yielding moment in SAP2000 is therefore taken equal to the cracking moment obtained from the *M-Phi* analysis. This approach facilitates the identification of most critical steps in the plastic hinge development. However, for pre-existing cracks where the concrete has no initial tensile strength, points B and C in Figure D-5 could be lumped into one point corresponding to the yield point of re-bars. The selected Points A to E in Figure D-5 correspond to similar points in SAP2000 used to define the hinge properties, as shown in Figure D-6.

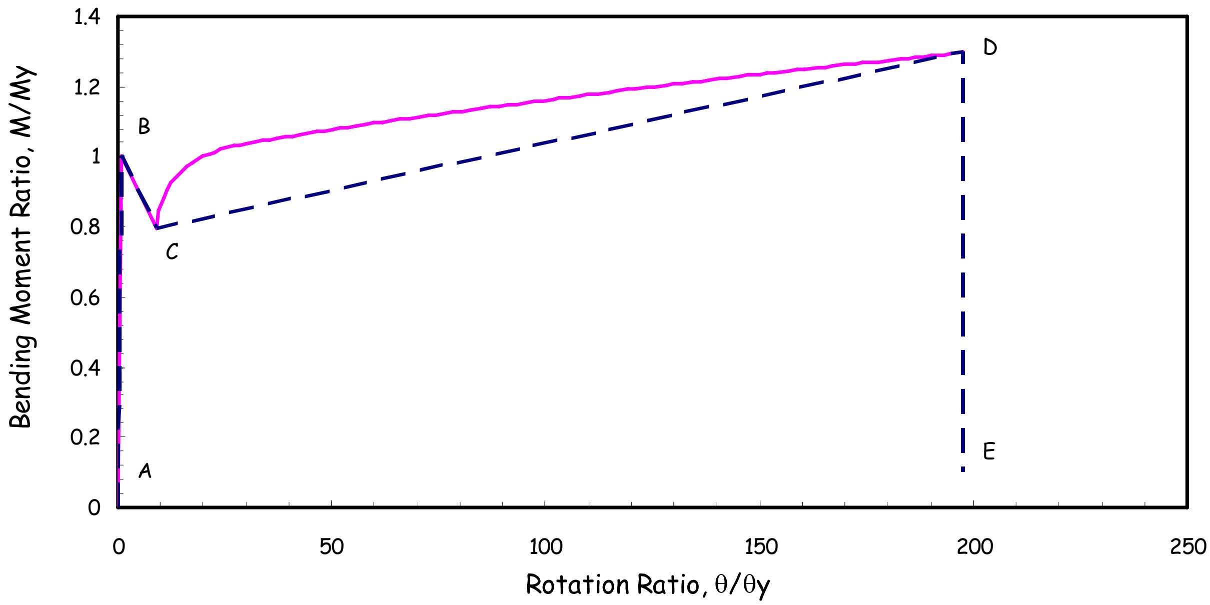
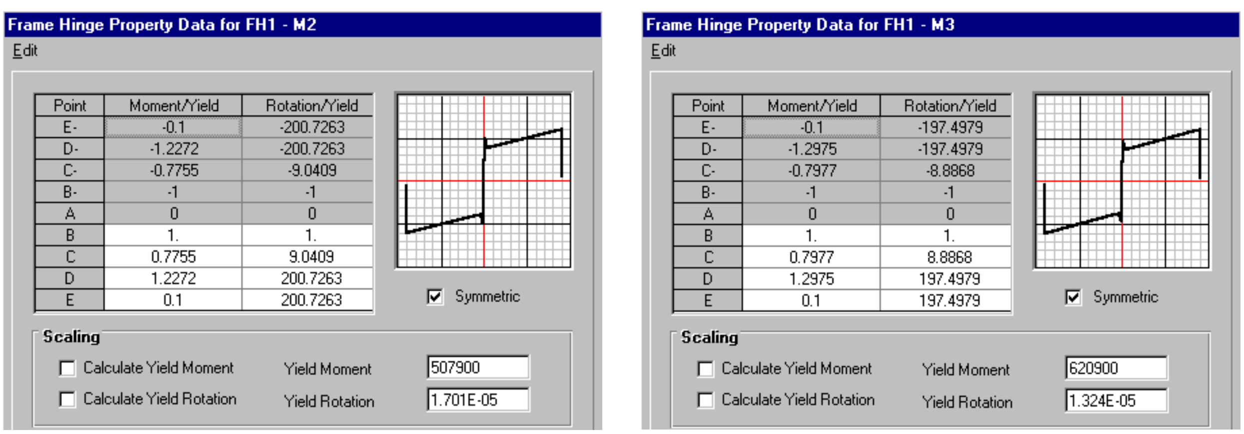


Figure D-5. Idealized moment-rotation diagram for SAP2000 input (M_y-θ_y)



M_x vs. θ_x M_y vs. θ_y

Figure D-6. Frame hinge property setup in SAP2000

(4) *Summary of SAP2000 pushover analysis*. In summary the following steps are required to perform static pushover analysis using SAP2000 program:

(a) Develop a structural model as you would do for any other analysis using frame and other types of elements. Assign material and section properties as needed.

(b) Perform the basic static analysis and, if desired, a linear-elastic analysis to check the model and compute mode shapes that may be used in defining pushover load patterns.

(c) Define hinge properties for frame elements corresponding to appropriate moment-rotation and force-displacement diagrams. Assign hinges at frame element ends or at any location along the element length, as appropriate.

(d) Define static pushover load cases (load pattern) that best describe the seismic inertia forces affecting the structure. For example, these may be taken proportional to the fundamental mode shape.

(e) Perform static pushover analysis.

(5) *Results of pushover analysis.* The results generated by SAP2000 include both pushover curves and capacity spectrum graphical representation. The pushover curves for the example tower are shown in upper graphs of Figure D-7 for loading along the longitudinal (x) and transverse (y) axes. The results for the capacity spectrum method for loading in the x and y directions are presented in the lower graphs of Figure D-7. The capacity spectrum method is an approximate nonlinear static procedure that predicts the inelastic displacement demand of the structure by combining structural capacity obtained from a pushover analysis with seismic demand represented by response spectra (ATC-40). In capacity spectrum method the demand response spectra and pushover curve are displayed in terms of spectral acceleration vs. spectral displacement. The resulting diagram shows a family of demand response spectra (shown as red) at various level of damping, a capacity spectrum (shown as black), and a family of straight lines representing constant periods (shown as green). The graph also includes a single demand spectrum with variable damping (shown as blue) whose intersection with the capacity spectrum gives the Performance Point, a point corresponding to the expected inelastic displacement demand of the structure. It is interesting to note that for this example the ratio of the inelastic displacement to the yield displacement is about 2, an indication that the performance of the tower beyond the performance point will be marginal because ductility ratio for the lightly reinforced example tower is expected not to exceed two.

(6) *Advantages of SAP2000 pushover analysis*

(a) Excellent pre-processing capabilities

(b) Built-in default plastic hinge properties for building structure based on FEMA-273 recommendations. However, user should define plastic hinge properties for lightly reinforced concrete structures

(c) Load patterns are easy to create

(d) Axial and shear failures along with bending failures can be considered, if desired

(e) Plastic hinges can be located at any point along the length of a frame element

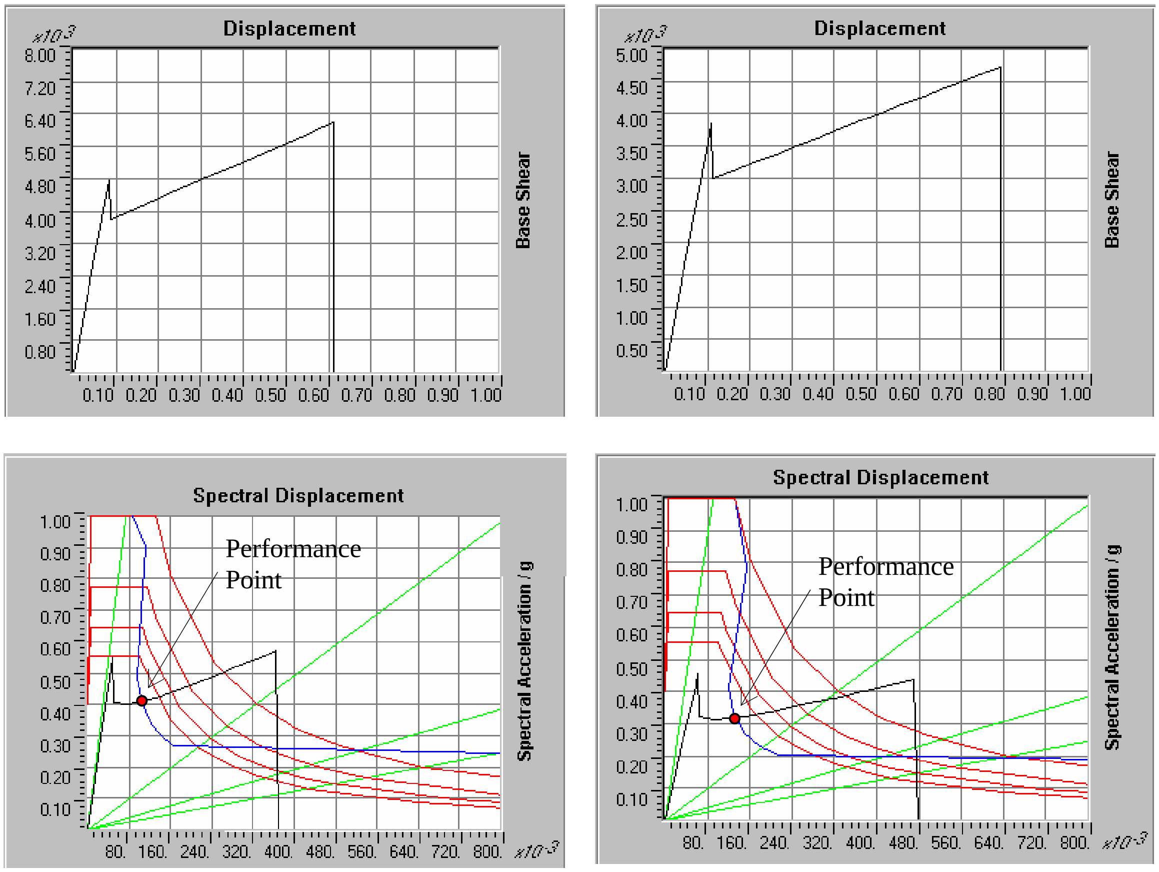
(f) Rupture of the re-bar or the breakage of the connection is determined

(7) *Disadvantages of SAP2000 pushover analysis*

(a) Plastic hinges are available for beam elements only

(b) The approximate location of plastic hinges should be known prior to the analysis

(c) Nonlinear link elements behave linearly during the pushover analysis.



Pushing in X-X direction Pushing in Y-Y direction

Figure D-7. Base shear vs. top displacement (above) and demand-capacity spectra (below)

d. Pushover Analysis using DRAIN-2DX program

(1) DRAIN-2DX is used to illustrate the application of a general nonlinear structural analysis program to push-over analysis. Three different element types are used for comparison purposes to show their strengths and shortcomings. These include: plastic hinge beam-column element (type 02), simple connection element (type 04), and fiber beam-column element (type 15). Each of these is briefly described below.

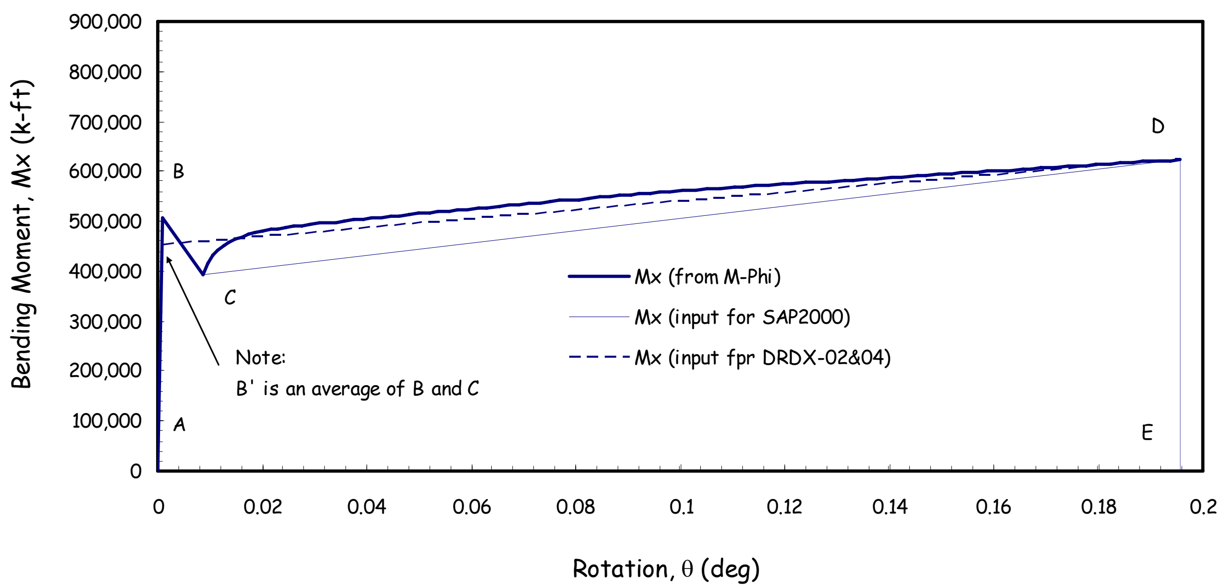
(2) *Plastic hinge beam-column element (type 02).* This element uses plastic hinge formation in bending to represent the nonlinear behavior. The plastic hinges are lumped at the element ends. The input for the plastic hinges includes an idealized bilinear moment-rotation relationship, which is defined by an initial stiffness, yield strength, and the post-yield strain hardening ratio. The idealized bilinear relationship may be obtained from approximation of an actual moment-rotation curve estimated using "*M-Phi*" program. Figure D-8 shows a possible approximation of the *M-Phi* moment-rotation curve, in which the yield strength B is obtained as an average of B and C and strain-hardening portion by connecting B to D. Also shown on this figure is the idealized moment-rotation relationship for SAP2000 program (i.e. ABCDE). The main advantage of the element is its ease of use, and disadvantages include:

(a) Element remains linear in axial and shear

(b) Plastic hinges form only at the element ends and have no length

(c) Since strain hardening is modeled by placing an additional parallel element, the stiffness of the parallel element should be estimated carefully so that it results in a correct strain-hardening ratio

(d) Requires moment-rotation relationship to be known prior to the analysis. In this example, it was obtained from "M-Phi" analysis.



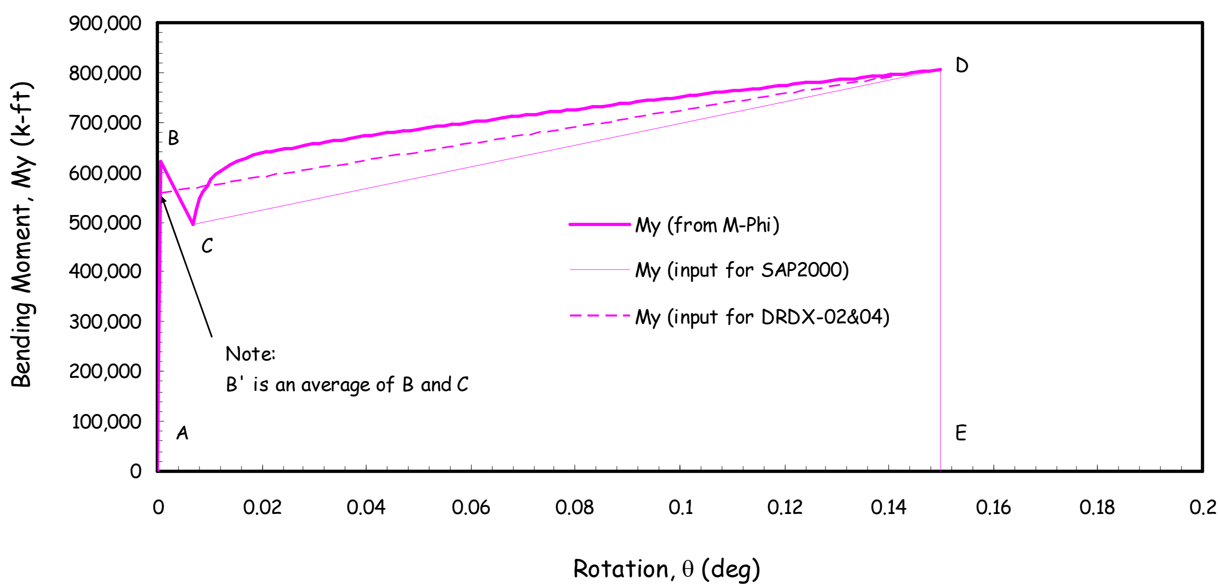


Figure D-8. Idealization and comparison of moment-rotation for SAP2000 and Drain-2D

(3) *Simple connection element (type 04).* Simple connection element allows the user to insert a nonlinear hinge and gap type connection at any location in the structure. The hinges can behave in tension, compression, bending, or shear deformation. One connection element for each type of deformation is permitted. The connection element is idealized by a bilinear relationship defined by an initial stiffness, yield force or moment, and the post yield stiffness ratio. The idealized bilinear moment-rotation for the connection element is obtained similar to that described for the Element Type 02 and shown in Figure D-8. The connection element has advantages of the ease of use and the ability to model axial and shear failure modes in addition to the bending. However, the use of additional elements with additional nodal points, prior knowledge about the possible locations and types of plastic hinges needed, as well as prior knowledge of the moment-rotation or force-displacement relationships are some of the drawbacks.

(4) *Fiber beam-column element (type 15).* Fiber beam-column element is more comprehensive than the Type-02 and Type-04 elements. The numerical models of the tower are developed by dividing the tower into a number of fiber elements. Each fiber element, in turn, is divided into several segments. The fiber element section consists of numerous fibers, each with its own material properties. In this example, the fiber element at the bottom of the tower where nonlinear behavior is expected is divided into a total of 95 fibers, 48 concrete fibers, and 47 steel fibers, as shown in Figure D-9. Note that for pushing in the y-direction fiber elements are developed parallel to the x-axis. Generally, more fibers increase the accuracy of the analysis; however the time of the computation also increases. Each fiber element is identified by its distance from the neutral axis and its section area. For example in Figure D-9 the hatched concrete fiber is located 6.55m (21’-6”) from the y-axis and has a section area of 4.46 m^2 (48 ft^2). Similarly, the farthest steel fiber along the positive x-axis is located 7 m (23 ft) from the y-axis with a section area of 0.036 m^2 (0.39 ft^2) for the 36 #11 bars located at this distance. The basic fiber element used in this example assumes full bond between the concrete and the reinforcing steel. The material model for the fiber element consists of the uni-axial stress-strain relationship for the concrete and steel fibers. No force-displacement or moment-rotation relationship is needed. The assumed material properties for the concrete and steel are given in Table D-1 and the corresponding stress-strain curves are shown in Figure D-10. The pushover analysis proceeded with the displacement control and the tower model was subjected to increasing lateral load until a lateral displacement of 30.48 cm (1 foot) was achieved.

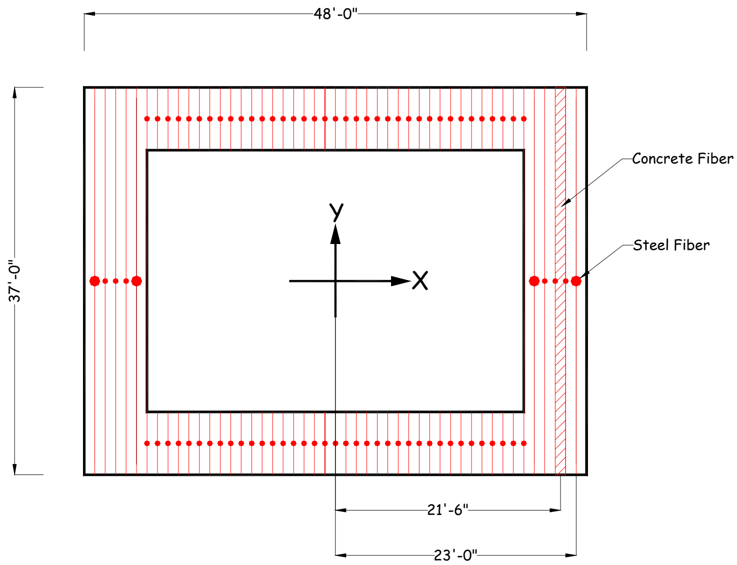


Figure D-9. Fiber element idealization of tower section for pushing in x direction

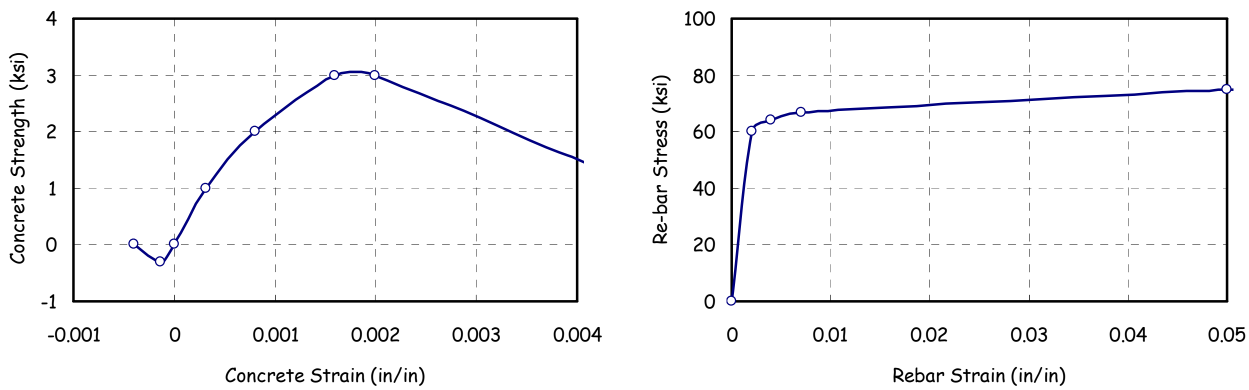


Figure D-10. Assumed stress-strain curves for concrete and reinforcing bar

(5) *Advantages of DRAIN-2DX fiber element (type 15)*

(a) Plastic hinges can be developed at any location in the element and plastic deformations are distributed along the element length and through the element cross-section.

(b) Plastic hinges have a final length.

(c) Re-bar bond-slippage and concrete crack opening and closing can be considered.

(d) Concrete strength degradation is considered.

(e) Reinforcing bar strength degradation can be considered.

(f) Concrete cracking, concrete crushing and re-bar yielding are identified in the output.

(g) No moment-rotation or force-displacement relationship is required.

(h) Developed strains and curvatures of the plastic hinge are directly output and can be plotted and evaluated.

(6) *Shortcomings of DRAIN-2DX fiber element*

(a) Relatively complicated to use.

(b) Very sensitive to material and section properties.

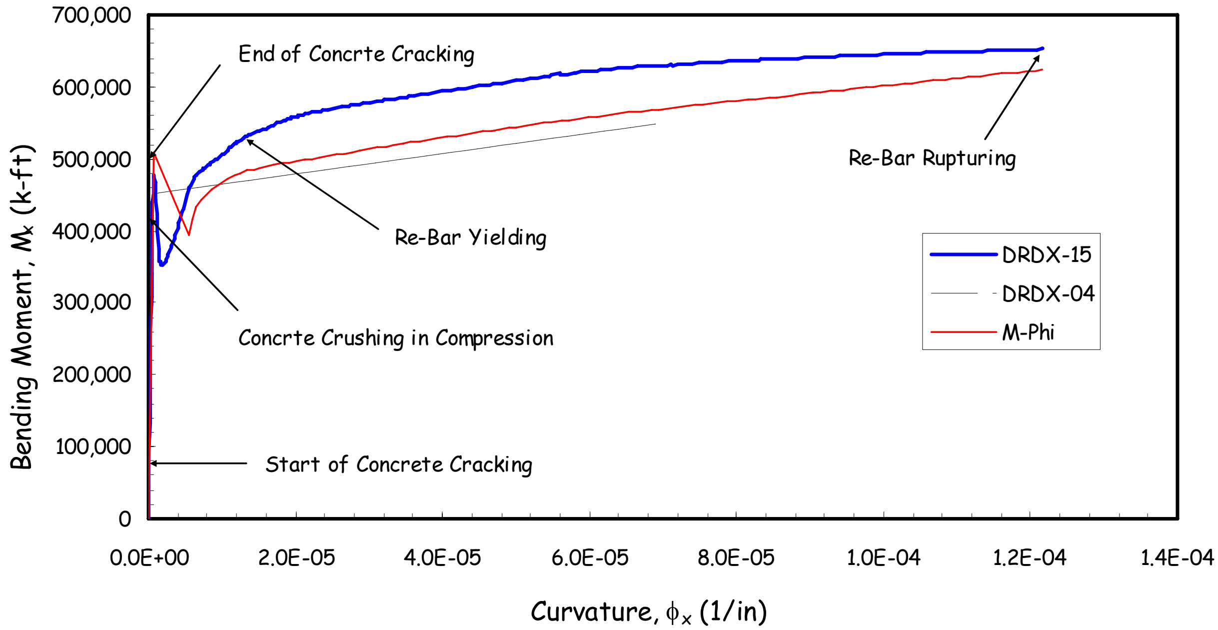
(c) Element remains linear in shear.

D-9. Evaluation and comparison of results

a. The results indicate that the inelastic response primarily occurs at the base of the tower. At this location the inelastic response starts with concrete tensile cracking followed by concrete crushing in compression prior to complete cracking of the section and yielding of steel reinforcements.

b. Moment Curvature. Figure D-11 shows moment-curvature relationships for bottom section of the tower computed for bending with respect to x and y axes. The figure compares moment-curvature relationships obtained from the *M-phi* and DRAIN-2D fiber element analyses. The figure also includes the idealized bending moment-curvature used in the DRAIN-2D analysis utilizing the connection element Type 04. The results for bending along x-axis (top graph of Figure D-11) indicate a cracking moment of about 687,400 kN-m (507,000 k-ft) for *M-Phi* and 650,793 kN-m (480,000 k-ft) for the fiber element. The concrete cracking is followed by a sudden drop in moment, but it is picked up as the load is fully transferred to the reinforcing bars. The moment drop caused by concrete cracking is more abrupt for the fiber element than it is for the *M-phi* analysis. Similarly, for bending with respect to y-axis, a cracking moment of about 840,608 kN-m (620,000 k-ft) was obtained from M-phi and slightly higher than 786,375 kN-m (580,000 k-ft) for the fiber element analysis (lower graph of Figure D-11). Again the moment drop due to concrete cracking is more abrupt for the fiber element than it is for the *M-phi* analysis. Overall there is a good agreement between the fiber element and *M-phi* moment-curvature analysis. The slight difference between the two models is due to the effects of shear and axial force inherent in the fiber model but not included in the *M-phi* calculation. The moment-curvature relationship in Figure D-11 is a measure of the local damage. The acceptance of local damage can be determined by comparing the induced inelastic curvature (or rotation) with the ultimate curvature capacity of the section. The ultimate curvature or rotation capacity of a section is computed according to the procedure outlined in Section D-7. Note that the moment-curvature in Figure D-11 was developed assuming that bond between the concrete and reinforcing steel will not fail and that post-elastic deformation of the steel can develop fully. If this bond is not strong and the bond slippage can occur, then the tower will fail prematurely before inelastic deformation of the reinforcing steel is fully realized. Bond slippage can be modeled to account for this failure mode but requires a special modeling technique which is beyond the scope of this example.

c. Pushover Curve. The global response of the tower as a plot of the base shear versus the lateral top displacement of the tower is shown in Figure D-12. In this figure the top graph is for loading along the strong x-axis and the lower graph for loading along the weak y-axis. These graphs summarize pushover curves for one SAP 2000 analysis and three DRAIN-2D analyses with three different types of elements. Generally, the results for all models are in good agreement up to completion of the concrete cracking and differ beyond this point where the inelastic deformation and yielding of reinforcing steel begin. In general the fiber element is better suited for capturing the inelastic response behavior and provides a more accurate pushover curve than the other models. At about 25 mm (1 inch) of lateral displacement along the x-axis (top graph of Figure D-12), the tower fully cracks at the bottom on the tension side, followed by transfer of tensile forces from the cracked concrete to reinforcing bars and crushing of the concrete on the compression side. The concrete crushing and load transformation occur at a slightly lower strength and continue without resistance for a lateral displacement of about 15 mm (0.6 inches), at which point the strength increases as the reinforcing bars are fully engaged in carrying the load (plateau portion of the DRDX-15 curve in upper graph of Figure D-12). Note that the basic fiber element used in this example assumes full bond between the concrete and reinforcing bars, thus the pullout or bond slip observed in the laboratory tests (Dove, 1998) were not included in this example. If the bond slip had been considered. the resulting pushover curve would have shown strength deterioration rather than strength gain. The pushover curve for loading in the weak direction shows similar behavior, except that force demands are lower and displacements are higher. For this example, the performance of the tower is considered acceptable if displacement ductility is limited to 2.5, which translates into a global displacement of about 70 mm (2.75 inches) in the strong direction and 89 mm (3.5 inches) in the weak direction. Note that these ultimate displacements are much smaller than the 360 mm (14.16 in.) predicted by the simplified displacement method.



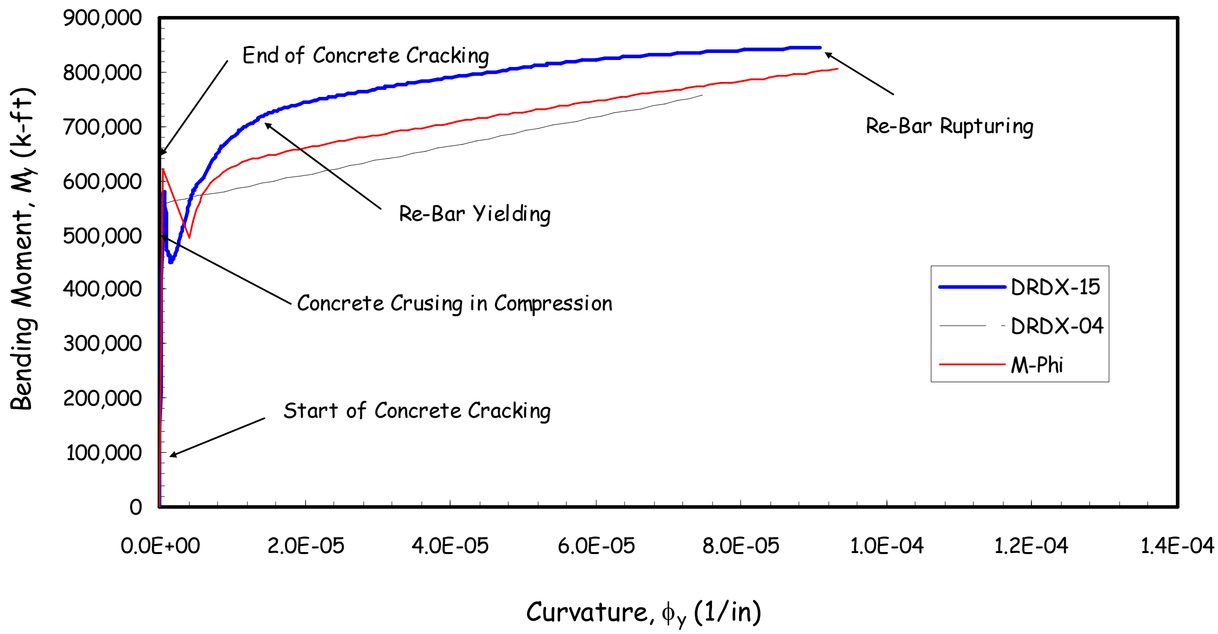
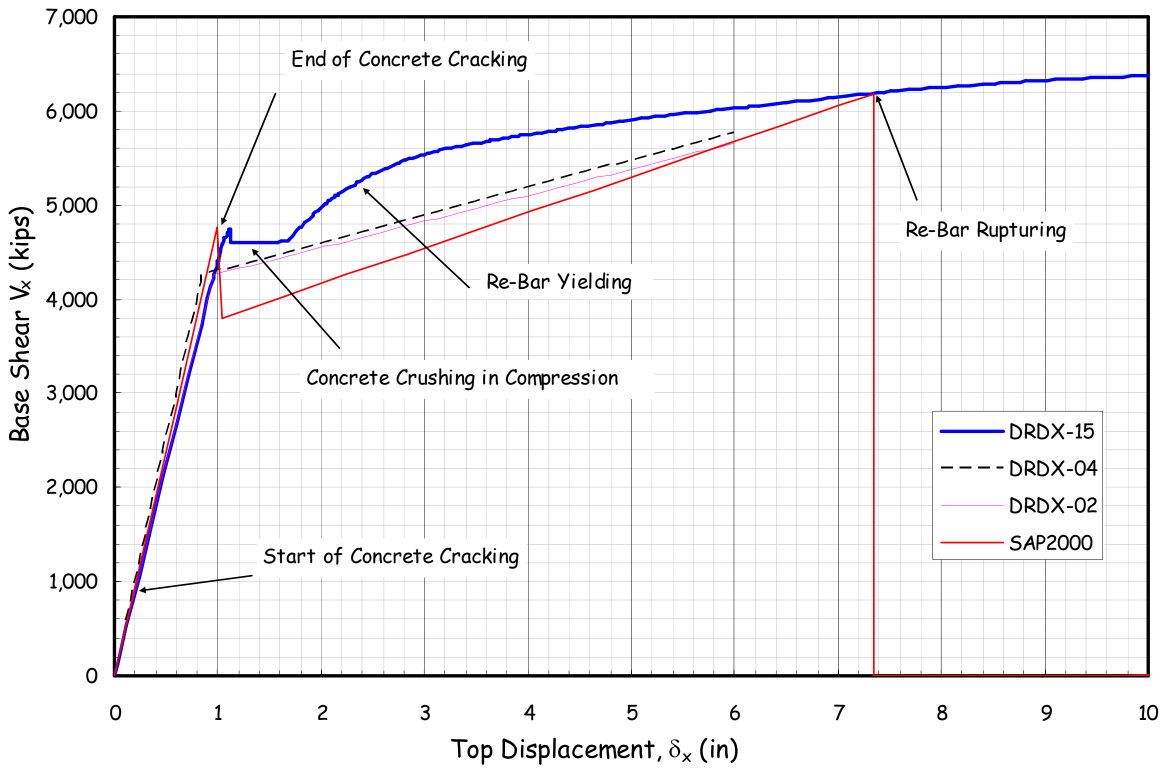


Figure D-11. Comparison of moment-curvature relationships obtained through different procedures



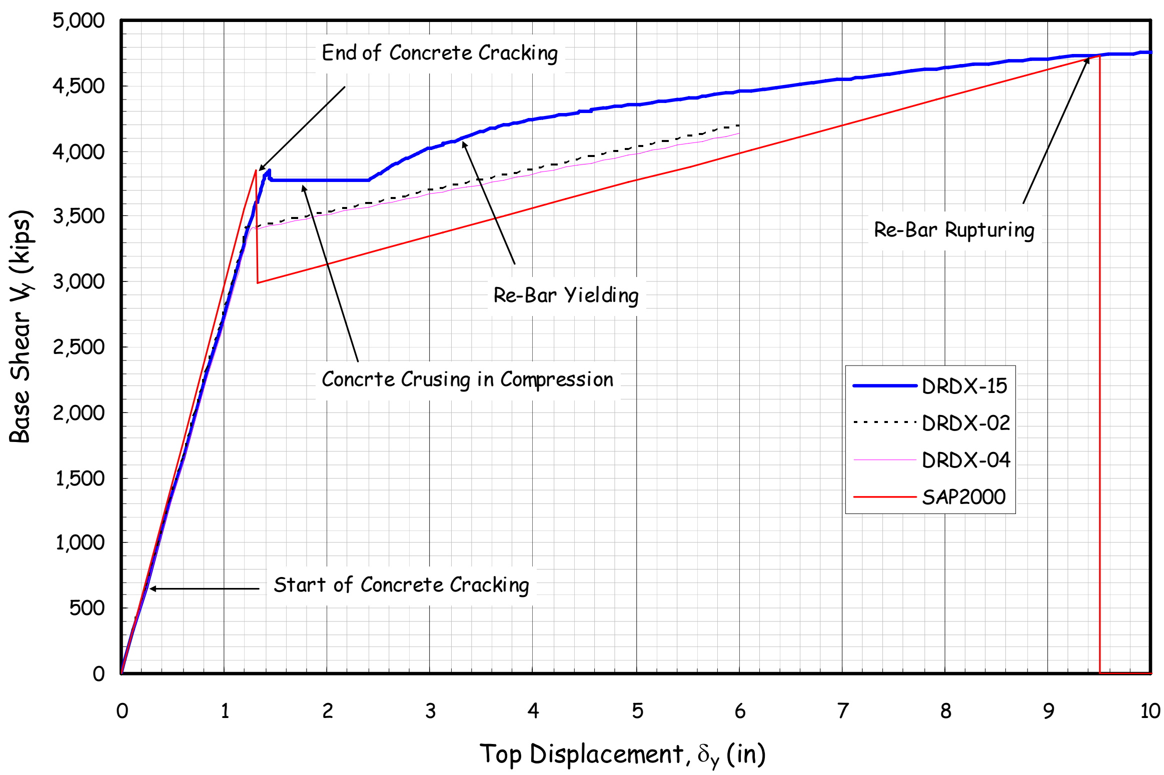


Figure D-12. Comparison of pushover curves from different procedures

D-10. Summary and Conclusions

Four different nonlinear models were used to illustrate nonlinear pushover analysis procedures for the example intake tower. They included a SAP2000 model with plastic-hinge frame element and three DRAIN-2DX models with lumped-plastic hinge (Type 02), connection hinge (Type 04), and fiber (Type 15) elements. The effectiveness and shortcomings of each model were briefly discussed. SAP2000 has excellent graphics capabilities and also provides spectrum capacity analysis. However, plastic hinges can be used only with frame elements and gap elements cannot be used in static analysis to model crack opening and separation. DRAIN-2DX element Type 02 permits plastic hinges to form at the end of beam-column elements only. The element's behavior is linear with respect to axial load and shear and nonlinear with respect to bending. The nonlinear behavior is represented by an idealized bilinear moment-rotation relationship, which is defined and input by the user. DRAIN-2DX element Type-04 allows the user to insert a nonlinear hinge and gap type connection at any location in the structure; the nonlinear hinges can model axial, bending, or shear deformation. Finally, DRAIN-2DX fiber element can be used that allows distributed plastic hinge formation at any location in the element. No moment-rotation or force-displacement relationship is required. Only uni-axial stress-strain curves for various materials are needed as the input. Overall, the fiber element provided more accurate results, except that strength degradation due to bond slip needs to be also modeled. The results showed that at a lateral displacement of about 25 to 35 mm (1 to 1.4 inches), the tower first fully cracks at the bottom on the tension side, followed by transfer of tensile forces from the cracked concrete to reinforcing bars and crushing of the concrete on the compression side. The concrete crushing and load transformation occur at a slightly lower strength and continue without resistance for a lateral displacement of about 15 to 25 mm (0.6 to 1 inch), at which point the strength increases if the concrete and reinforcing steel are fully bonded, otherwise strength degradation will occur due to bond slippage. Based on the results of this example, it appears that a displacement ductility of 2.5 is appropriate for lightly reinforced towers but should not be exceeded.